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Analytical fragility curves for typical Algerian reinforced concrete bridge piers

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Abstract. This paper illustrates the results of a seismic vulnerability study aimed to derive the fragility curves for typical Algerian reinforced concrete bridge piers using an analytical approach. Fragility curves express the probability of exceeding a certain damage state for a given ground motion intensity (e.g., PGA). In this respect, a set of 41 worldwide accelerometer records from which, 21 Algerian strong motion records are included, have been used in a non-linear dynamic response analyses to assess the damage indices expressed in terms of the bridge displacement ductility, the ultimate ductility, the cyclic loading factor and the cumulative energy ductility. Combining the damage indices defined for 5 damage rank with the ground motion indices, the fragility curves for the bridge piers were derived assuming a lognormal distribution.

Keywords: analytical fragility curves; damage index; bridge piers; strong motion records

1. Introduction

Seismic vulnerability assessment and development of fragility curves for existing bridges are a matter of great concern among the researchers in the recent years (Kurian *et al.* 2006, Seongkwan *et al.* 2007, Padgett and DesRoches 2008, Moschonas *et al*. 2009). Fragility curves of bridges can be developed empirically as well as analytically. Empirical fragility curves are usually developed based on the damage reports from past earthquakes. When actual bridge damage and ground motion data are not available, analytical fragility curves can be used to assess the performance of bridges (Choi *et al.* 2004, Nielson and DesRoches 2007). In Algeria, neither bridge damages nor their performances have been reported during the previous earthquakes that have struck the country, aside from those observed during the 2003 Zemmouri earthquake. According to the ASCE post earthquake investigation report (ASCE 2004), the most significant bridge damages were due to the superstructure moving off their bearings and dropping onto the bents caps, columns damage

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observed in old bridges, shear key at some metallic girders, superstructure rotation, vertical movement, girder movement and buckling as well as some damage at seat-type abutment and to bin-type wingwall. Nevertheless, the bridges have performed well and no long interruption of their serviceability was noticed during the earthquake. In this respect, due to the lack of information from past earthquakes damage on bridges, it is not possible to derive fragility curves empirically for the typical bridge piers in Algeria. Therefore, fragility curves have been developed analytically from non linear dynamic analyses of typical bridge piers. Since damage states are mostly related to structural capacity (C) and the ground motion intensity parameter is related to structural demand (D), the probability of failure (p_f) gives the probability that the seismic demand will exceed the structural capacity. Mander and Basoz (1999) have presented the theory of fragility curves for highway bridges based on uncertainties in various bridge parameters to evaluate seismic vulnerability of typical bridges. While Ghobarah *et al.* (1997) have quantified numerically the damage states from the dynamic responses of the bridges under various levels of ground motion excitation; Hwang *et al*. (2001) described a detailed procedure for analytical development of fragility curves.

The main objective of this study is to develop analytical fragility curves for typical Algerian reinforced concrete bridge piers based on a numerical approach taking into account, the structural parameters and the variation of the input ground motion. Prior to the newly established Algerian seismic regulation code for bridge structures (RPOA 2008), the bridge piers have been designed using the seismic design coefficient method. In this respect, seismic coefficients equal to 10% of the total weight in the horizontal direction and 7% of the total weight in the vertical direction have been used to design the bridge piers. By using worldwide strong motion records, the damage indices as defined by Park and Ang (1985) are obtained through a non-linear dynamic response analysis via the educational NONLIN software program (Charney 1998). The obtained damage indices defined for five damage rank and the ground motion indices are then combined to derive the corresponding fragility curves for the reinforced concrete bridge piers.

2. Methodology to develop analytical fragility curves

This part describes the methodological steps used to construct the analytical fragility curves for some specific Algerian reinforced concrete bridge piers. As stated above, the piers were designed using the simplified seismic design method for bridges in Algeria. In this respect, the yield stiffness of the piers was firstly obtained by performing a sectional static analysis by means of the RESPONSE 2000 computer program (Bentz and Collins 2000). For the non- linear dynamic response analysis, the piers were modelled as a single degree of freedom (SDOF) system and subjected to a total of 41 acceleration time histories taken from a worldwide earthquake data base based on their peak ground acceleration values. For the non linear dynamic response analysis, the PGA of the selected records was normalized to different excitation level from 0.1 g to 1.0 g having 10 excitation levels with equal intervals. Using these acceleration time histories as an input motion, the Park-Ang damage indices of the bridge piers are obtained from the non linear analysis. Finally, the obtained damage indices and the corresponding ground motion indices are combined to develop the analytical fragility curves for the RC bridge piers. The schematic diagram for constructing the analytical fragility curves (Karim and Yamazaki 2001) is shown in Fig. 1.

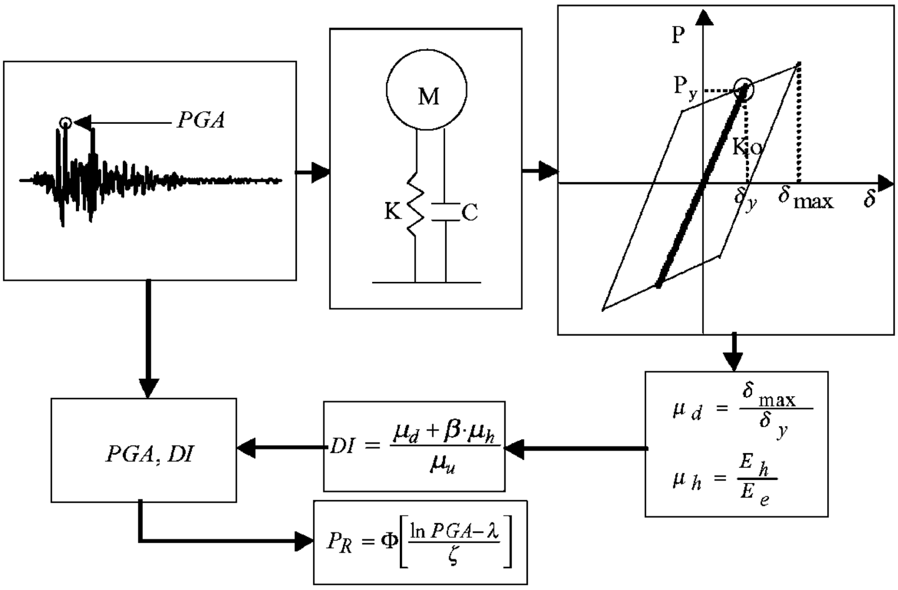


Fig. 1 Schematic diagram for constructing the fragility curves for RC bridge piers

3. Static analysis

The sectional analysis is carried out for two reasons: (1) to find out the two possible structural failure modes, i.e., the shear or the flexural failure modes of the bridge piers and (2) to obtain the force-displacement relationships at the top of the bridge piers. In the case of a flexural analysis, there is no contribution of the shear component to the displacement, however, in the case of a shear analysis, there is contribution of both the shear and the flexural components to the displacement. The displacement at the top of the bridge pier is given by the following equation.

$$\delta = \sum_{i=1}^{N} (\phi_i \times dy \times d_i + \gamma_i \times dy) \tag{1}$$

where δ is the displacement at the top of the bridge pier, N is the number of its cross-sections, ϕ_i is the curvature of the section i, dy is the width of each cross-section of the pier, d_i the distance from the top of the pier to the centre of gravity of each cross section and, γ_i is the shear strain.

4. Dynamic analysis

To perform the dynamic response analysis, the piers are modeled as a single-degree-of freedom (SDOF) system using a bilinear model (Priestley *et al.* 1996). The damage assessment of the bridge piers is carried out using the Park-Ang damage index *DI* expressed as

$$DI = \frac{\mu_d + \beta\mu_h}{\mu_u} \tag{2}$$

Table 1 Relationship between the damage index (*DI*) and damage rank (*DR*)

Damage index (*DI*)	Damage rank (*DR*)	Definition
0.00 < DI ≤ 0.14	D	No damage
0.14 < DI ≤ 0.40	C	Slight damage
0.40 < DI ≤ 0.60	B	Moderate damage
0.60 < DI ≤ 1.00	A	Extensive damage
1.00 ≤ DI	As	Complete damage

where μ_d is the displacement ductility, μ_u is the ultimate ductility of the bridge piers, β is the cyclic loading factor taken as 0.15 and μ_h is the cumulative energy ductility defined as

$$\mu_h = \frac{E_h}{E_e} \tag{3}$$

with E_h and E_e denoting the cumulative hysteretic (obtained from dynamic analysis) and the elastic energy (obtained from elastic analysis) of the bridge piers respectively. The damage indices of the bridge piers are obtained using Eq. (2). The obtained damage indices for the given input ground motion are then calibrated to get the relationship between the damage index (DI) and the damage rank (DR). This calibration is performed using the Ghobarah *et al*. (1997) proposed method. Table 1 shows the relationship between the damage index and the damage rank. As it can be seen, each DR has a certain range of DI varying from no damage to complete damage. Using the relationship between DI and DR, the number of occurrence of each damage rank is obtained. These numbers are then used to obtain the damage ratio for each damage rank.

5. Numerical examples

Some manageable number of typical structural bridge piers in Algeria has been selected for the fragility analysis, considering four typical RC bridge piers. As it deals with piers that are not designed according to the 2008 new Algerian seismic design code for bridges (RPOA 2008), it is assumed that only the size and the reinforcement of the piers can be changed with other conditions such as their height, the length and the weight of the superstructure. The four sample bridges used to perform the analysis are listed in Table 2 and shown in Fig. 2. A brief description of these bridges is given here after:

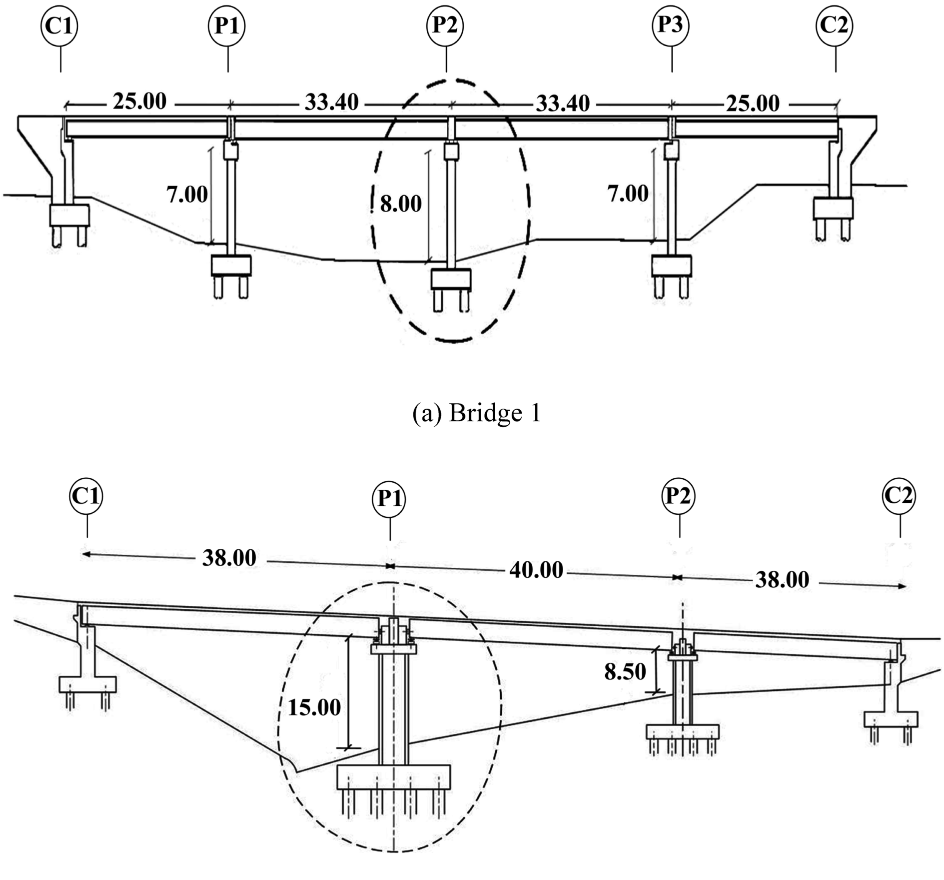
- Bridge 1 consists of a four spans with an overall length of 116.80 m. The superstructure consists of a longitudinally reinforced concrete deck slab of 10 m wide and it is supported by three sets of columns and by an abutment at each end. Each set has three columns with a circular cross section of 1.20 m diameter.
- Bridge 2 has an overall length of 116.00 m with three spans. It is supported by two hollow core concrete bridge piers of rectangular cross section having external dimension of 6.0 m × 3.5 m with a hollow core of 4.80 m × 2.30 m. The deck width is 15.70 m. The piers have a varying height with the taller one of 15.00 m and the shorter one of 8.50 m.
- Bridge 3 has an overall length of 64.20 m with two spans. It is supported by a wall pier type of a rectangular cross section having 8.61 m × 0.80 m dimensions and 6.81 m height. The deck

width is 10.05 m.

- Bridge 4 has an overall length of 151.80 m with four spans. The bridge is supported by three hammerhead piers with a cross section of 5.00 m × 1.80 m. The deck width is 13.06 m. The piers near the abutments have 14.00 m in height while the central one has a height of 17.00 m.

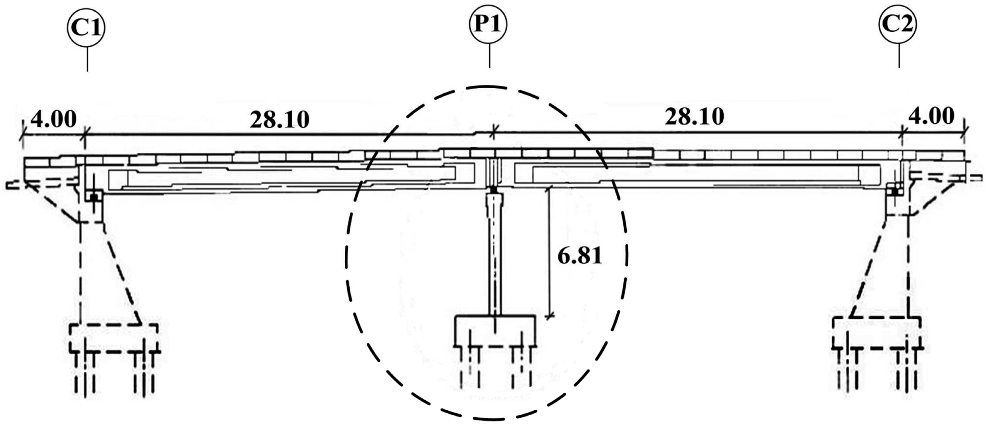
Table 2 Description of four sample bridges

Bridges	Overall length (m)	Number of spans	Deck width (m)	Column height (m)
1	116.80	4	10.00	7.00 - 8.00 - 7.00
2	116.00	3	15.70	15.00- 8.50
3	64.20	2	10.50	6.81
4	151.80	4	13.06	14.00 - 17.00 - 14.00

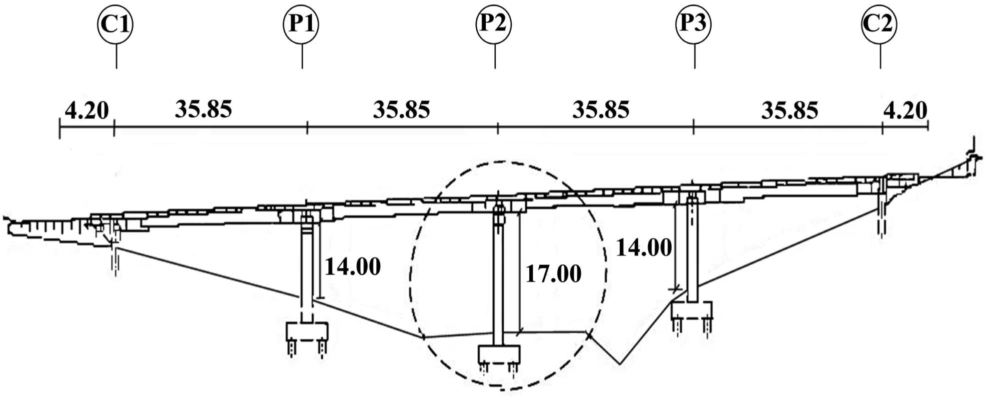


(b) Bridge 2

Fig. 2 Elevation of sample bridges



(c) Bridge 3



(d) Bridge 4

Fig. 2 Continued

6. Moment curvature curves for lateral direction

The sectional analysis of the bridge pier is carried out to get the moment curvature relationship necessary for the non linear analysis. In this respect, the cross sectional dimension of the pier bridge, the yield strength of steel σ_{sy}, the compressive strength of concrete σ_c', the diameter of the longitudinal reinforcement bars as well as the tie reinforcement bars are taken as input parameters. Figs. 3(a), 3(b), 3(c) and 3(d) show the cross sections and the resulting moment- curvature curves of the bridge piers. For the sectional analysis, the height of the pier bridge taken into consideration is: 8 m, 15 m, 6.09 m and 17 m respectively for bridge 1, 2, 3 and 4. It is found that in most cases, the flexural failure governs the failure mode.

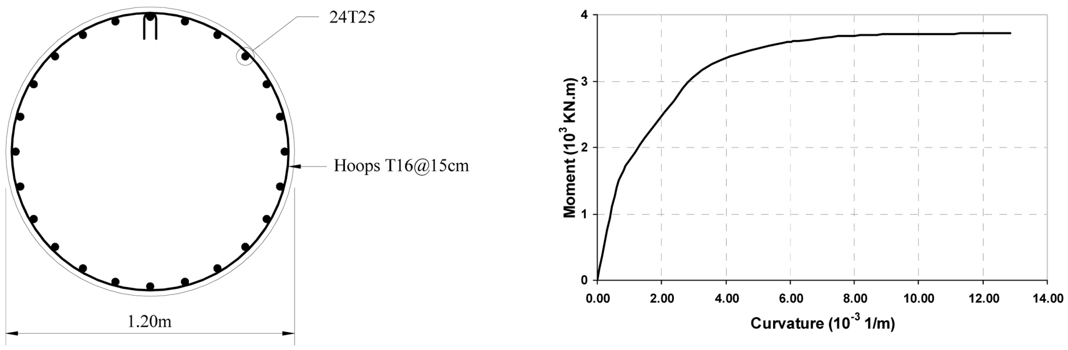


Fig. 3(a) Cross section and its moment curvature curve for the bridge pier (Sample bridge 1)

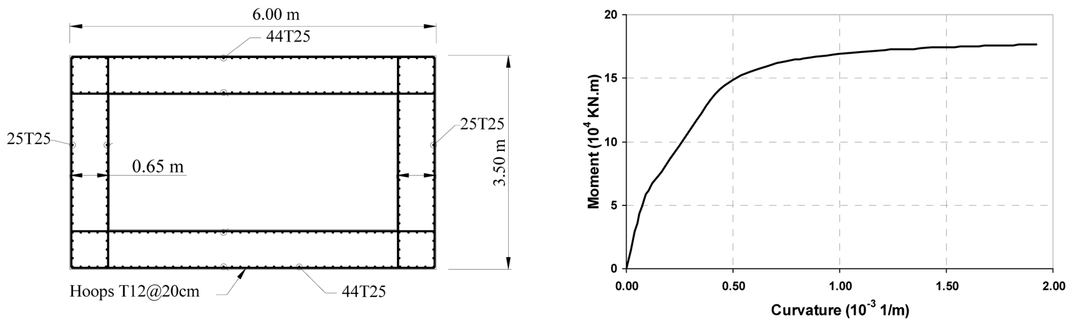


Fig. 3(b) Cross section and its moment curvature curve for the bridge pier (Sample bridge 2)

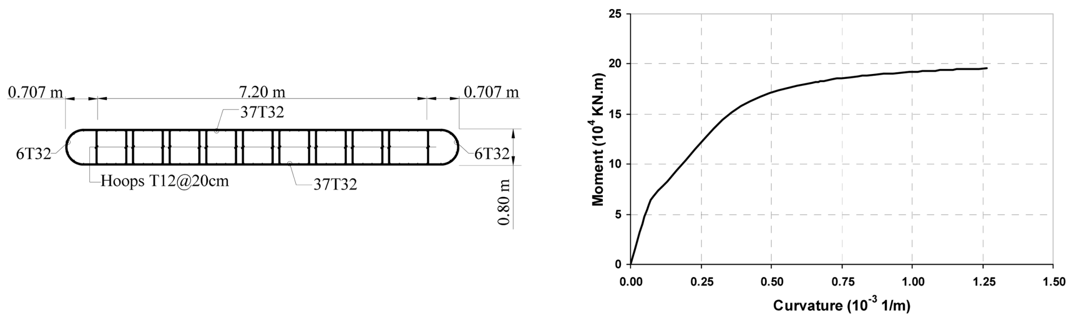


Fig. 3(c) Cross section and its moment curvature curve for the bridge pier (Sample bridge 3)

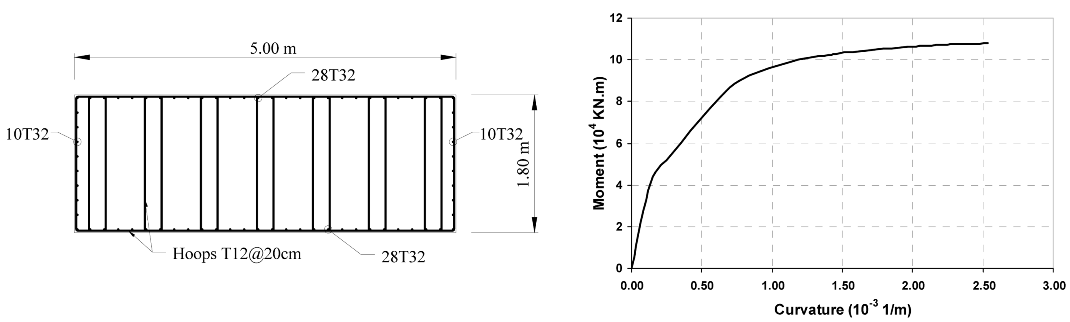


Fig. 3(d) Cross section and its moment curvature curve for the bridge pier (Sample bridge 4)

7. Fragility curves

Established fragility curves are constructed with respect to PGA. The damage ratio for each damage rank at each excitation level is obtained by calibrating the DI using Table 1. Based on this data, fragility curves for the bridge piers are derived assuming a lognormal distribution. The cumulative probability of occurrence P_R of a damage equal or higher than rank R is given as

$$P_R = \Phi\left[\frac{\ln X - \lambda}{\zeta}\right] \tag{4}$$

Where Φ is the standard normal distribution, X is the ground motion indices in term of PGA, The two parameters of the distribution λ and ζ are the mean and the standard deviation of ln X. The log-normal distribution has a probability density function

$$f(x, \mu, \sigma) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-\left(\frac{(\ln(x)-\mu)^2}{2\sigma^2}\right)} \tag{5}$$

Where x is the value at which the function is evaluated, μ is the median value of the PGA and σ is the log-standard deviation.

The cumulative log-normal distribution is obtained by integration of the area below the density function shown in Eq. (6).

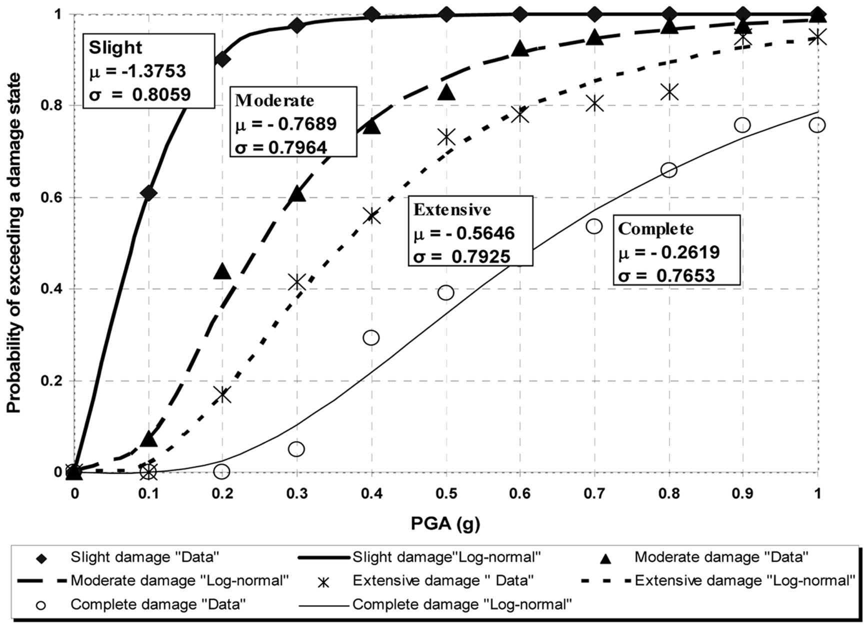
$$f(x, \mu, \sigma) = \frac{1}{x\sigma\sqrt{2\pi}} \int_0^x \frac{e^{-\left(\frac{(\ln(x)-\mu)^2}{2\sigma^2}\right)}}{t} dt \tag{6}$$

In order to obtain the two parameters that define the log-normal distribution (μ, σ), the Microsoft Excel Solver tool was used. Microsoft Excel applies the Generalized Reduced Gradient Nonlinear Optimization Code.

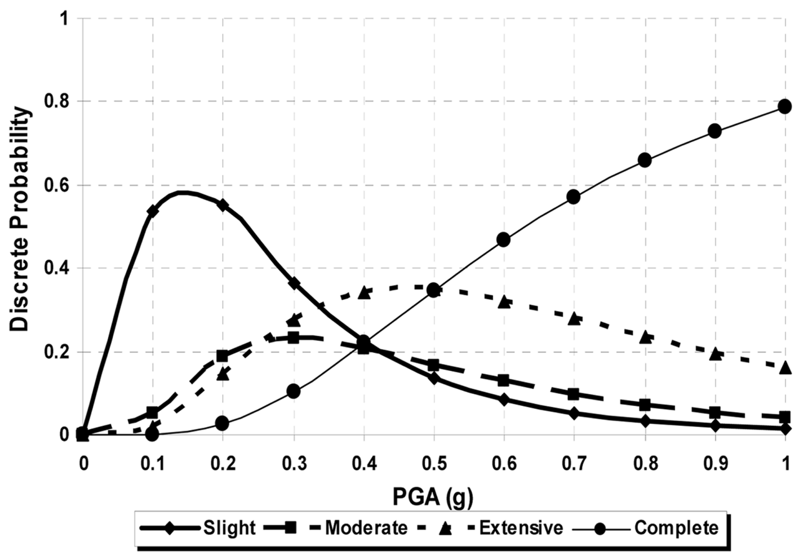
i. Define a preliminary value for the median and standard deviation (μ, σ);
ii. Plot the values obtained from the data;
iii. Calculate the cumulative log-normal distribution using the two preliminary values of μ and σ;
iv. Calculate the sum of the difference between the probability found from the lognormal probability plot constructed in step (iii) and the probability plot constructed in step (ii);
v. Perform the optimization code included in Microsoft Excel;
vi. Repeat this procedure for each damage state.

The Cumulative density functions (CDF), which should be converted to discrete damage-state probability curves are obtained by taking the difference in probability between adjacent damage state fragility curves. It means that the discrete slight probability is obtained by subtracting the slight damage state to the moderate damage state at each PGA value. The same step is done for each damage state curve. Figs. 4, 5, 6 and 7 show the fragility curves, for each damage state and for the entire sample pier bridges.

• Bridge 1:



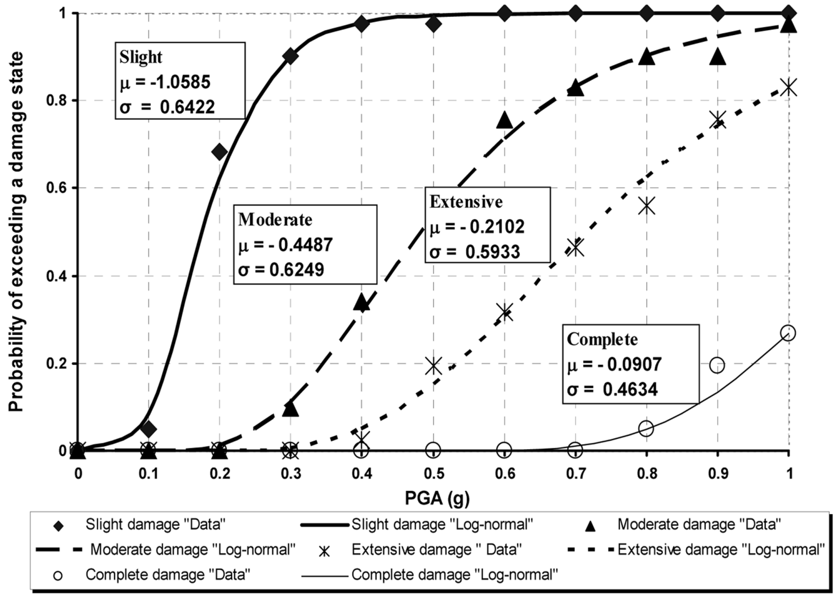
(a) Fragility curves for all damage states: Bridge pier's sample 1



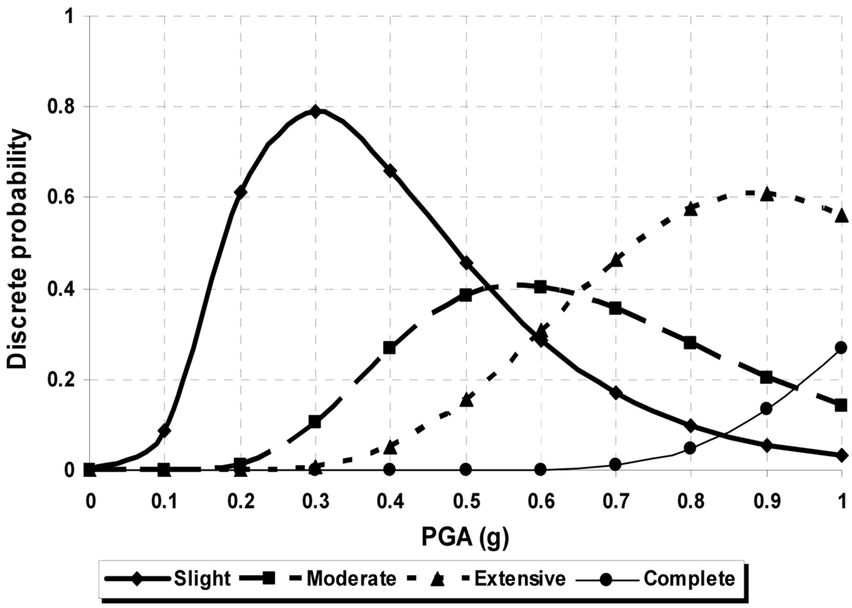
(b) Discrete damage state probability curves: Bridge pier's sample 1

Fig. 4 Fragility curves for the bridge pier's sample 1

• Bridge 2:



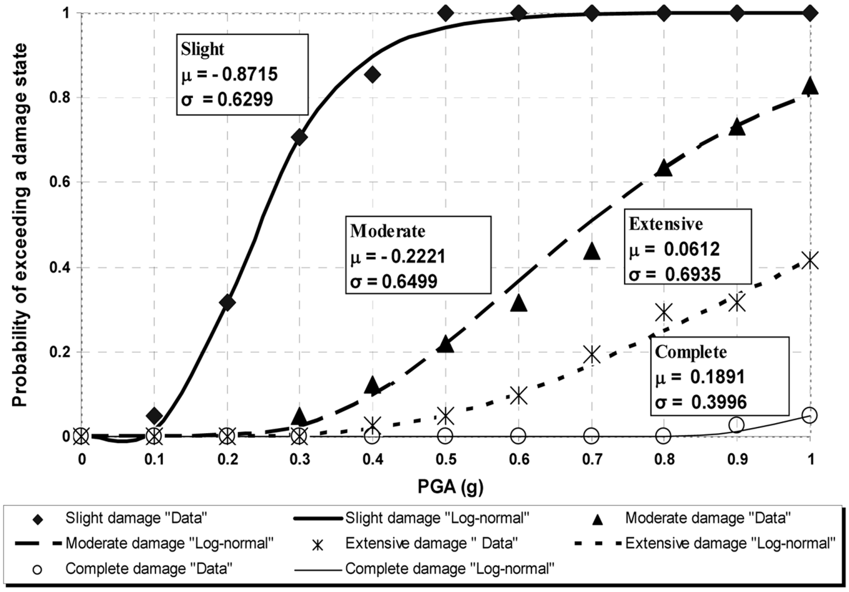
(a) Fragility curves for all damage states: Bridge pier's sample 2



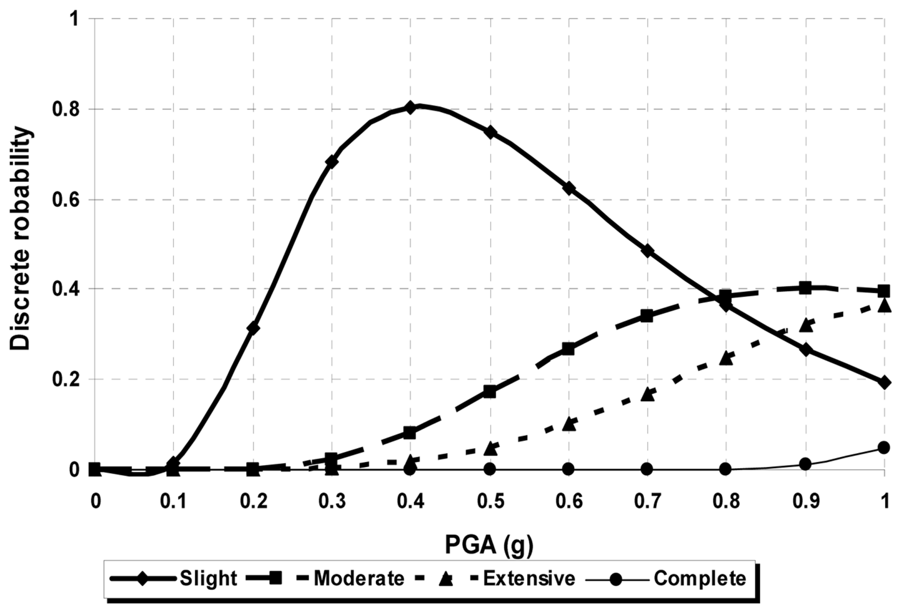
(b) Discrete damage state probability curves: Bridge pier's sample 2

Fig. 5 Fragility curves for the bridge pier's sample 2

• Bridge 3:



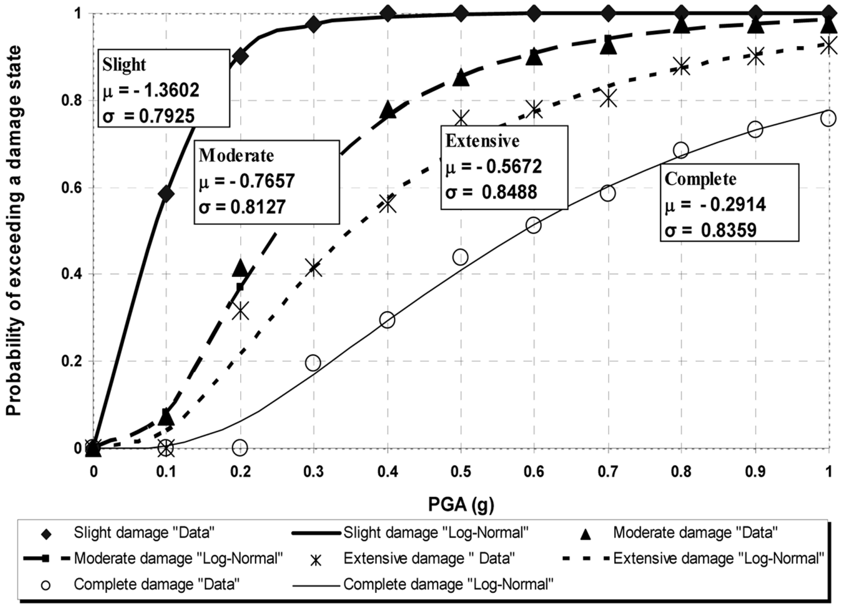
(a) Fragility curves for all damage states: Bridge pier's sample 3



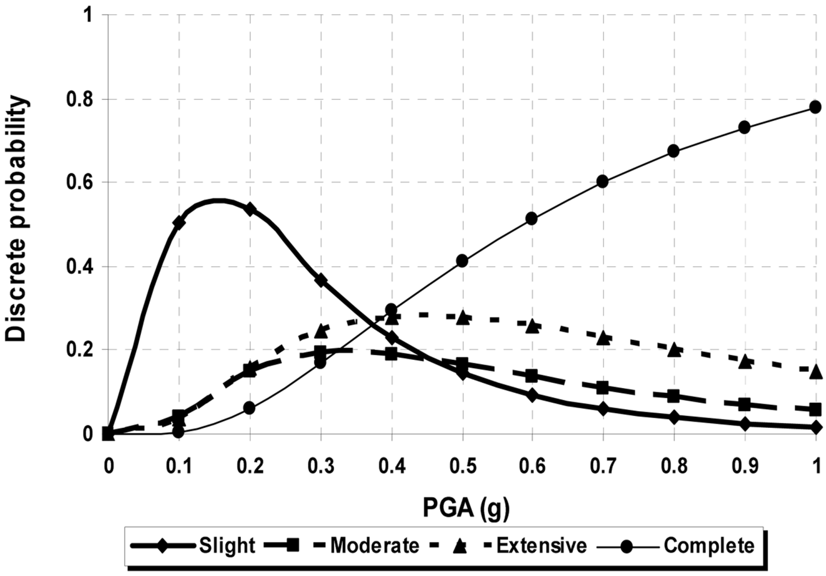
(b) Discrete damage state probability curves: Bridge pier's sample 3

Fig. 6 Fragility curves for the bridge pier's sample 3

• Bridge 4:



(a) Fragility curves for all damage states: Bridge pier's sample 4



(b) Discrete damage state probability curves: Bridge pier's sample 4

Fig. 7 Fragility curves for the bridge pier's sample 4

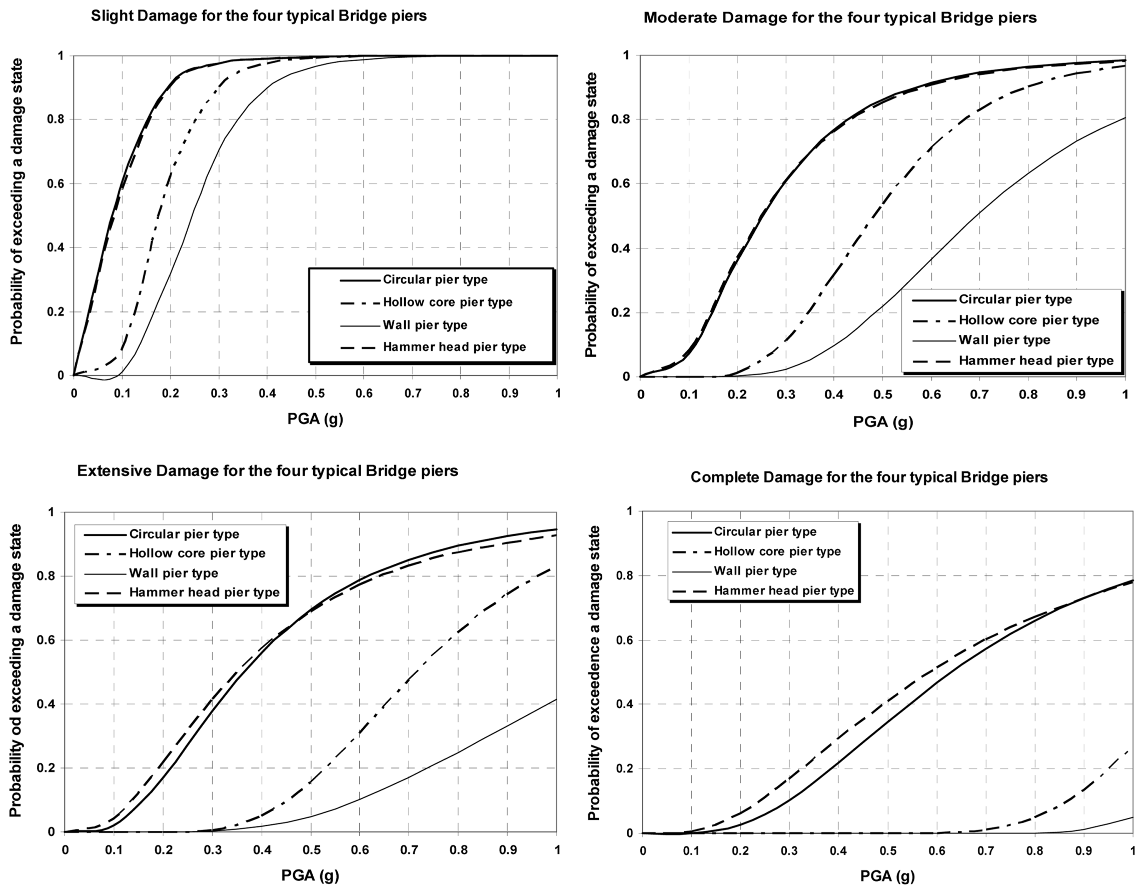


Fig. 8 Fragility curves between slight, moderate, extensive and complete damages for the four typical bridge piers

Fig. 8 shows the comparison between slight, moderate, extensive and complete damages for the four typical RC bridge piers.

8. Results interpretation

Analytical fragility curves for four typical Algerian reinforced concrete bridge piers having different structural properties (Fig. 3(a), Fig. 3(b), Fig. 3(c) and Fig. 3(d) were obtained with respect to the peak ground acceleration based on numerical simulation using 41 worldwide accelerometer records assuming a lognormal distribution. It was found that there is a significant effect on the fragility curves due to the variation of structural parameters in terms of the cross section shapes, the longitudinal reinforcement and the tie reinforcement. The level of damage probability in the cases of *slight, moderate, extensive* and *complete damage* is the same for bridge type 1 (circular pier type) and bridge type 4 (hammerhead pier type). The bridge type 3 (wall pier type) has a lower level of damage probability than the other ones. However, the level of damage probability for the bridge type 2 (hollow core pier type) is lower than bridges type 1 (circular pier type) and 4 (hammerhead pier type) but higher than the bridge type 3 (wall pier type). It implies that the bridge type 3 which is supported by a wall pier type performs better against seismic forces than the others. The same observation can be done for the bridge type 2 that is supported by two hollow core piers type which performs better than the bridge type 1 (circular pier type) and bridge type 4 (hammerhead pier type).

9. Conclusions

To predict the extent of probable damages of bridge structures, fragility curves are regarded to be a useful tool. The vulnerability assessment of bridges is useful for seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of highway transportation systems. Using the analytical approach developed by Karim and Yamazaki (2001) for typical Japanese bridge piers, this paper illustrates the results of the seismic vulnerability study aimed to develop the analytical fragility curves for typical Algerian bridge piers based on numerical simulations. Bridge piers designed with the simplified seismic design method for bridges in Algeria are analyzed, and a large number of worldwide accelerometer records from which, Algerian strong motion records and earthquake records from some major event, e.g., the 1995 Kobe, the 1994 Northridge were selected in order to get a wide range of the variation of input ground motions. The fragility curves for the bridge piers are then developed by performing both, the static and the non linear time history analyses and following the same numerical approach that is described in chapter 2. One pier model has been selected as a representative of all other piers for a particular bridge structure. It can be seen that the analytical fragility curves for the four bridge piers show a very different level of damage probability with respect to PGA. This difference is due to the shape of the cross section and the percentage of the longitudinal and tie reinforcements. The wall pier type shows the best seismic performance while compared to the others (circular pier, hollow core pier and hammer head pier). The effect of soil-structure interaction is not taken into account for deriving the analytical fragility curves, for which a further study is also necessary.

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